TECHNISCHE UNIVERSITÄT BRAUNSCHWEIG

SONDERFORSCHUNGSBEREICH
BRANDVERHALTEN VON BAUTEILEN

ECCS 4(G)
13.4.84 Luxemburg

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# Calculation of critical temperatures of fire-exposed steel columns



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Frachtsendungen:
Braunschweig-Hauptgüterbahnhof
Erfüllungsort und Gerichtsstand Braunschweig

## Calculation of critical temperatures of fire-exposed steel columns

For a complete theoretical analysis of the load-bearing characteristics of fire-exposed steel columns, all the deformation and stress factors have in view of the effects of the theory of the 2nd order (geometrical non-linearity) to be considered. The strains that apply to load bearing elements without longitudinal restraint comprise thermal strains, strains resulting from stress, and strains resulting from warm creep factors (fig. 1). A great number of publications neglect this warm creep term, and the stress-dependent strains are determined from bilinear or non-linear stress-strain relationships established in steady-state tests. Some American as well as Swedish publications consider the warm creep term determined on the basis of the Hamarthy creep theory. According to Hamarthy, this warm creep term is subject to the stress as well as the temperature history. This implies that for a numeric analysis, the entire fire history needs to be considered in small steps of time, which leads to time consuming and numerically complicated methods of calculation.

In the Recommendations of the ECCS, however, stress-strain relationships are defined on the basis of transient-state tests, which allow the time of collapse or a collapse temperature to be determined directly. The entire problem can thus be subdivided into a calculation of load-bearing capacity and a simple calculation of heating response. Transient-state tests are carried out at a constant stress and a constant heating rate. So, these tests do not consider any stress history, and, strictly speaking, they only apply to the heating rate as defined.

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Tests carried out by Sonderforschungsbereich 148 "Brandverhalten von Bauteilen" of Technische Universität Braunschweig (SFB 148) with unprotected steel columns at a heating rate of approx. 50 °C/min., and protected steel columns at a heating rate of 3 - 10 °C/min., load application being centric as well as eccentric, revealed that different heating rates or variations in stress caused by column deflections do not have any significant effect on the collapse temperature. A simplified approach for the determination of the collapse time of fire-exposed steel columns, be they protected or unprotected, by applying stress-strain relationships derived from transient-state tests therefore seems to be admissible.

Results obtained from transient-state tests however do show a more or less significant influence of the heating rate on deformability (fig. 2). But when comparing the results produced by different material researchers, the influence of test-specimen cross sections on the deformability in transient-state tests becomes apparent. Temperature-strain curves, and, as a consequence, stress-strain relationships get "stiffer" as the specimen cross-section increases. At the same time, the influence resulting from different heating rates significantly decreases with an increasing specimen cross-section. Generally speaking, this confirms the results obtained from full-scale tests on steel columns, and it also provides an explanation for the fact that when checking column tests by calculations, the stress-strain relationships of the ECCS Recommendations obtained from small specimen proved to be too"soft" (fig. 3). The stress-strain relationships arrived at with transient-state tests carried out by SFB 148 with larger specimen, also proved to be too "soft" for various temperature ranges.

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Comparative calculations on fire-exposed steel columns made from HE-sections with hinge joints and a defined, equal load eccentricity at both ends (standard bar) always revealed the same typical strain pattern when buckled about the weak axis, irrespective of the formulation of the temperature-dependent material behaviour (fig. 4). Subject to slenderness and loading rate, the strains of the less affected flange were, up to the point at which the column failed, within the elastic compression or tension range. The inner, more affected flange, however, with the only exception of extremely slender and at the same time highly loaded columns, always showed strains in the semi-plastic range, which at small slenderness ratios and small loads were in the order of 5 - 8 $^{o}/oo$. An HE-section loaded in this way can be compared to a sandwich cross-section. The theoretical analysis shows that its failure is in the first place determined by the shape of the stress-strain relationship in the semi-plastic range. The temperature-dependent modulus of elasticity being given, comparative calculations with a suitable column test will thus allow a point on the stress-strain curve to be determined (stress-strain relationship from tests with structural members).

Among the fire tests carried out by SFB 148 with steel columns, a sufficiently large number of tests is suited for a determination, in the afore-mentioned manner, of the shape of stress-strain curves for temperatures between $150 \leq T \leq 700$ $^{o}$C. As expected, stress-strain relationships arrived at in the above manner are by far stiffer than those obtained with transient-state tests, and softer than those obtained with steady-state tests.

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The temperature-dependent modulus of elasticity (fig. 5) as well as the quasi-yield-stress at an expansion of 12 $^{o}/oo$ (fig. 6) coincide with the findings of a literature analysis made by Klingsch /1/, of an investigation carried out by the British Steel Corporation /2/, and they also correspond to the results of the transient-state tests carried out by Winkelmann at SFB 148 /3/. All the results, however, go significantly beyond the figures in the ECCS Recommendations /4/. Fire tests performed with steel columns in the past few years cover the entire spectrum of most diverse parameters. Investigated parameters are:

- slenderness
- direction of buckling
- structural shapes
- loading level
- load eccentricity
- support conditions (Euler cases II, III, IV)
- temperature gradient
- longitudinal restraint
- unprotected columns
- protected columns
- partially protected columns.

Check calculations made for all the steel-column fire tests performed by SFB 148, as well as by other institutions (approx. 100 tests), using stress-strain relationships derived from tests with structural members (fig. 7) revealed that test and calculation coincide convincingly. With the only exception of a few tests, where as a result of side effects as for instance torsional buckling there was an early failure, the deviation between computed collapse temperature and test-determined collapse temperature always is $\leq$ 20 $^{o}$C. In conjunction with the computer programme set up by SFB 148, this allows to reliably calculate the deformation and load-bearing behaviour of any type of steel column subjected to any thermal load.

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Figures 8 to 11 show a comparison of collapse temperatures established by an extensive theoretical analysis which considers the theory of the 2nd order, and collapse temperatures determined by a simplified method according to fig. r-4 of the ECCS Recommendations /4/. Hence it follows that this simplified method has to be regarded as a rather conservative assessment of the collapse temperature of fire-exposed steel columns.

Collapse temperatures may differ substantially for the two directions of buckling. This is due in particular to the fact that at temperatures above room temperature the residual stresses increasingly come to bear (figs. 12, 13). The significance of the residual stresses in turn is subject to the load level.

The constant steel temperature over the whole column length assumed in these calculations is an idealization which is not in conformity with a natural fire or a test fire under ISO conditions. Steel temperatures near the column ends are in fact more or less significantly lower than at mid column height. The favourable effect this has on the collapse temperature is negligible with columns with hinge joints at both ends (Euler case II), with rotationally restrained columns, however (Euler cases III and IV), the collapse temperature is raised considerably (fig. 14).

Although steel columns may very often be calculated according to Euler case II, they can because of the joints provided in case of a fire be regarded as partially or even completely rotationally restrained at both ends. The load level is thus reduced, which in conjunction with the low steel temperatures at both bar ends results, in particular in case of buckling about the weak axis, in a further rise of the collapse temperature (fig. 15). The largest portion of this temperature rise comes to bear at a rotational restraint at only one

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column end. This gives rise to the conclusion that even a partial rotational restraint at either one or both column ends, which is quite normal for conventional steel structures, has a clearly favourable effect on the collapse temperatures (also see /5/).

Among experts, the centrically loaded steel column which is rotationally restrained at either one or both ends (Euler cases III and IV) is very often considered to be best suited as test specimen when it comes to defining a practice-oriented collapse temperature. This is, however, not compatible with the certainly justified requirement that test results obtained with steel columns in a fire test should be reproducible and should permit a comparison with the findings of tests performed in other furnaces. Tests carried out in the past showed that because of imperfections which can never be excluded, centrically applied test loads effect a deformation of several millimeters at mid column height. Figure 16 illustrates with a series of tests carried out on 6 steel columns, which only differed in their yield stress at room temperature, the effect of deformations of 1 to 5 mm has on the collapse temperature. Subject to the loading level, there may be a test result scatter of up to 100 °C. For a protected steel column having a mean heating rate of t = 5 °C/min., this corresponds to a difference in time of 20 minutes.

It is for this reason that in the Federal Republic of Germany new standards were set for tests on fire-exposed steel columns, which took effect on 1st July 1983. The most essential modification is that, the rotational restraint at one column end being retained (Euler case III), the test load is applied at a defined eccentricity of approx. L/500, thus limiting the effects of imperfections on the test result to an acceptable degree. In view of the considerable influence the actual yield stress can have on the collapse temperature in case of small to mean slenderness ratios, the test result is subsequently

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modified by simple interpolation related to the guaranteed yield stress (for St 37 $\sigma_y$ = 240 N/mm$^2$).

With a view to real conditions and design loads according to German DIN Standard 4114 (fig. 17), the critical temperature of T = 500 $^{\circ}$C laid down by the German DIN Standard 4102 has to be regarded as a conservative assessment. A simplified determination of the collapse temperature of centrically loaded steel columns to fig. r-4 of the ECCS Recommendations /4/ by contrast leads to results which are by far more unfavourable and, consequently, by far more uneconomical.

It has been mentioned earlier that the heating rate does not influence the collapse temperature to any significant degree, and so the collapse temperatures found apply to any fire exposure (natural fires). The fire resistance time in natural fires can therefore be ascertained by calculating the equivalent fire duration, consideration being given to the temperature criterion (fig. 18).

## References


/1/ Klingsch, W.: Traglastberechnung instationär thermisch belasteter schlanker Stahlbetondruckglieder mittels zwei- und dreidimensionaler Diskretisierung. Dissertation, Technische Universität Braunschweig (1975).

/2/ British Steel Corporation: Supporting technical data to the British Steel Corporation proposed revision of chapter 2 of the European recommendations for the design of steel structures exposed to standard fire, September 1980.

/3/ Winkelmann, O.: Verformungsverhalten von Bau-, Beton- und Spannstählen bei hohen Temperaturen. Arbeitsbericht 1978/80 Teil II des Sonderforschungsbereichs 148 "Brandverhalten von Bauteilen" der Technischen Universität Braunschweig.

/4/ European Recommendations for the Fire Safety of Steel Structures. ECCS - Technical Committee 3 - Fire Safety of STeel Structures (1983).

/5) Witteveen, J.; Twilt, L.: A Critical View on the Results of Standard Fire Resistance Tests on STeel Columns. Fire Safety Journal, 4 (1981/1982).

I $\varepsilon_{total} = \varepsilon_{th(T)} + \varepsilon_{\sigma(T,\sigma)}$

stress-strain relationships
(steady state tests: T = const., $\sigma \neq$ const.)

II $\varepsilon_{total} = \varepsilon_{th(T)} + \varepsilon_{\sigma(T,\sigma)} + \varepsilon_{cr(t,T,\sigma)}$

warm-creep term
(Dorn, Harmathy)

III $\varepsilon_{total} = \varepsilon_{th(T)} + \varepsilon_{\sigma(T,\sigma)}$

stress-strain relationships
(transient state tests: T $\neq$ const., $\dot{T}$ = const., $\sigma$ = const.)

IV $\varepsilon_{total} = \varepsilon_{th(T)} + \varepsilon_{\sigma(T,\sigma)}$

thermal strain

stress-strain relationships
(from column tests: T $\neq$ const., $\dot{T} \neq$ const., $\sigma \neq$ const.)

Fig. 1: Strain components to describe the deformation behaviour of structures at elevated temperature

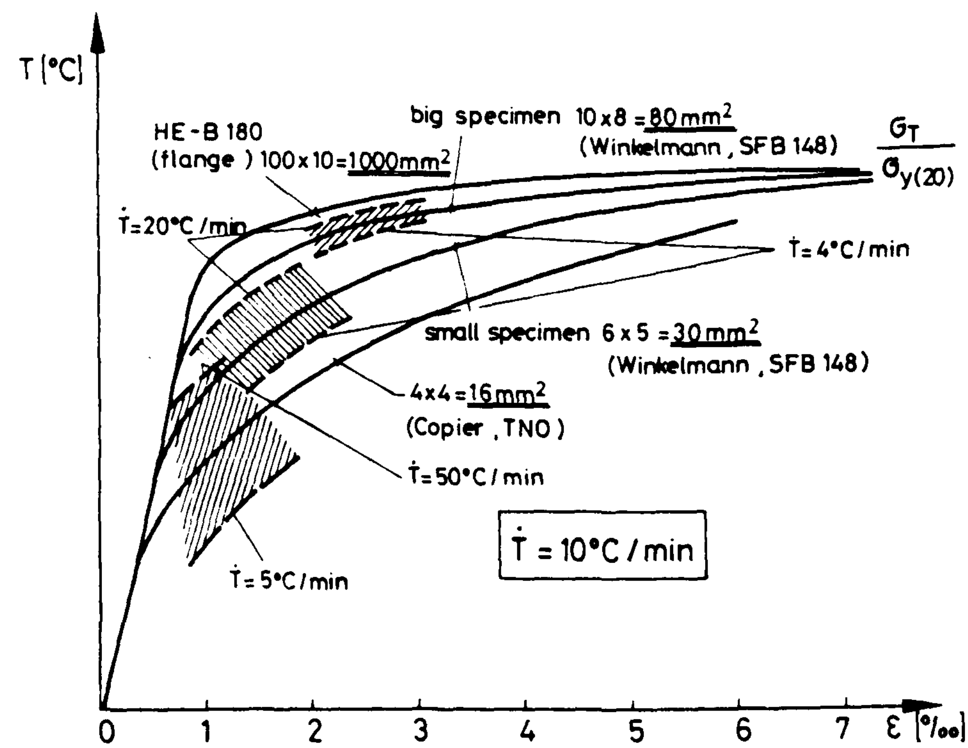



Fig. 2: Influence of dimension of test specimen and heating rate on strains of transient state tests

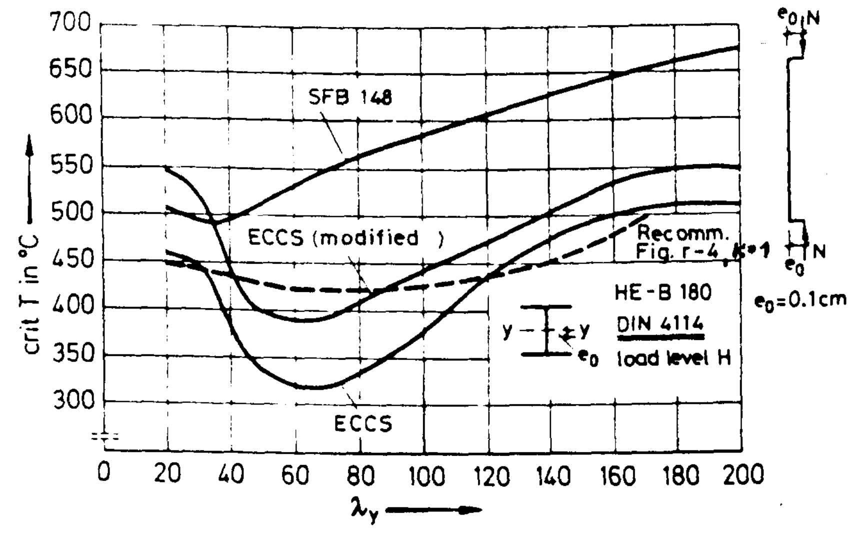



Fig. 3: Calculated critical temperature of steel columns using different stress-strain relationships

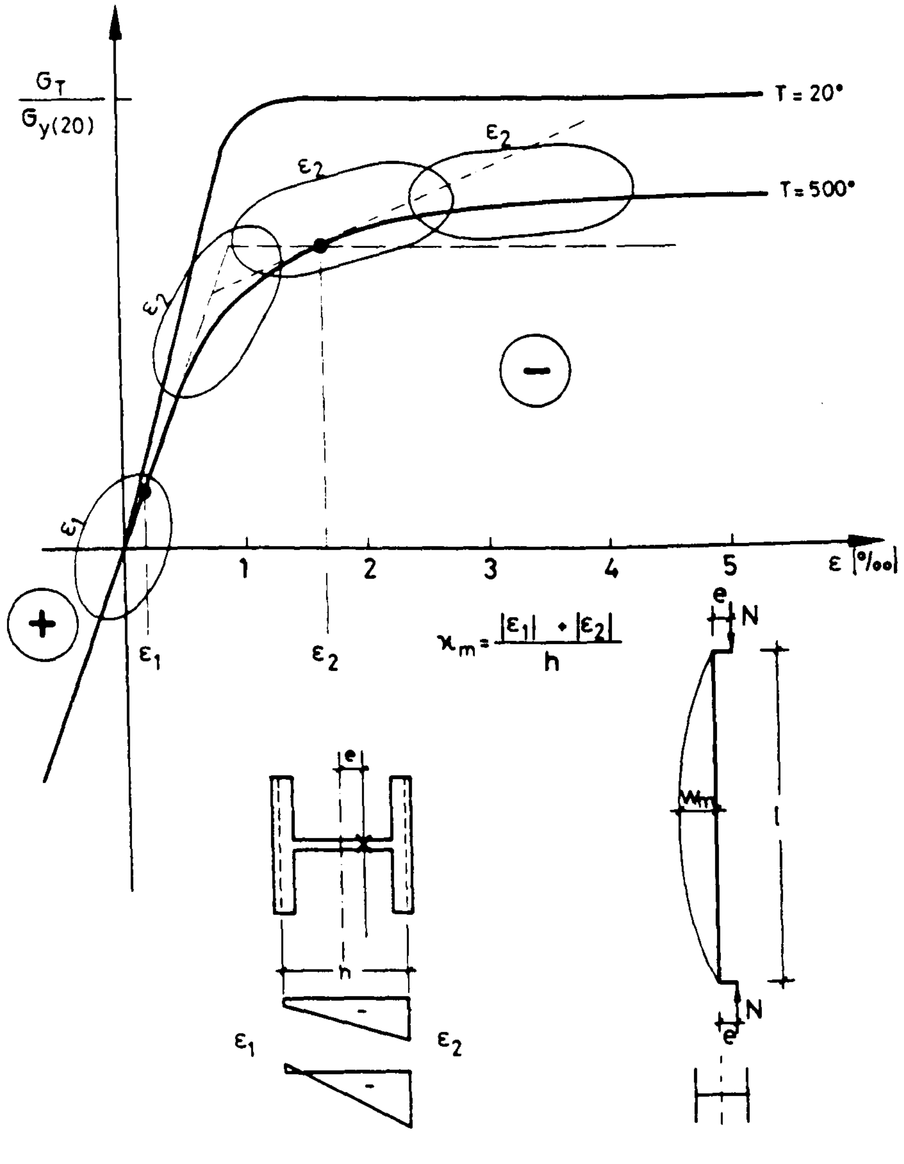



Fig. 4: Typical strains at failure time of steel columns (HE-sections) under fire action

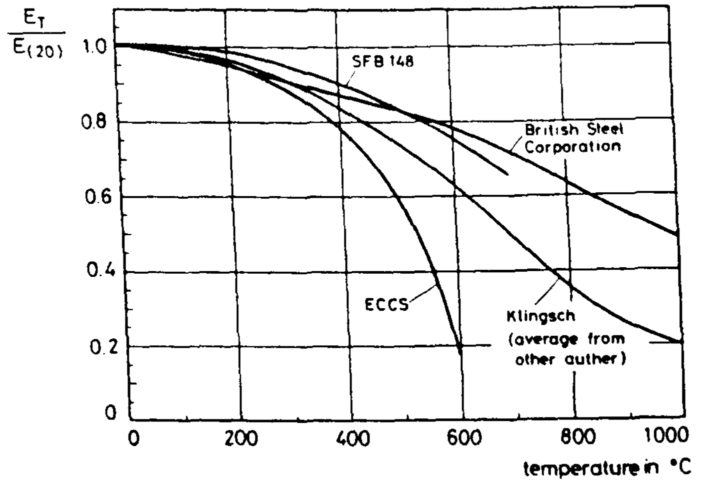



Fig. 5: Modulus of elasticity (Young's Modulus) as a function of the steel temperature

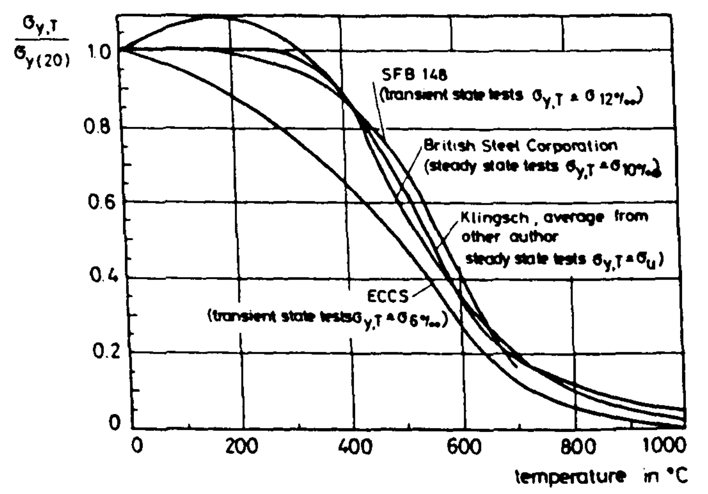



Fig. 6: Yield stress as a function of the steel temperature

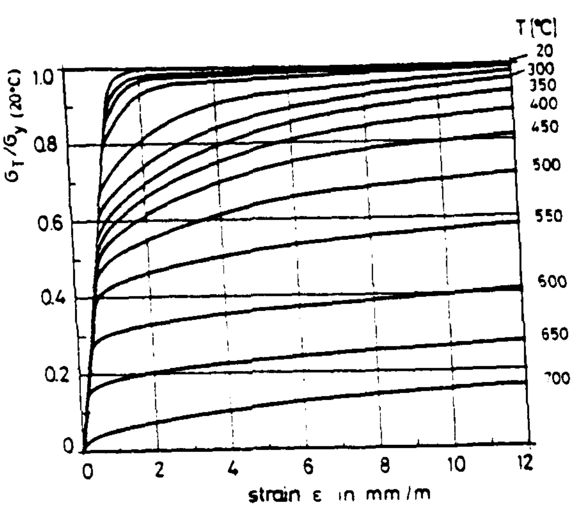



Fig. 7: Stress-strain relationships at elevated temperatures of a structural steel ($\sigma_{y,20}$ = 240 N/mm²)

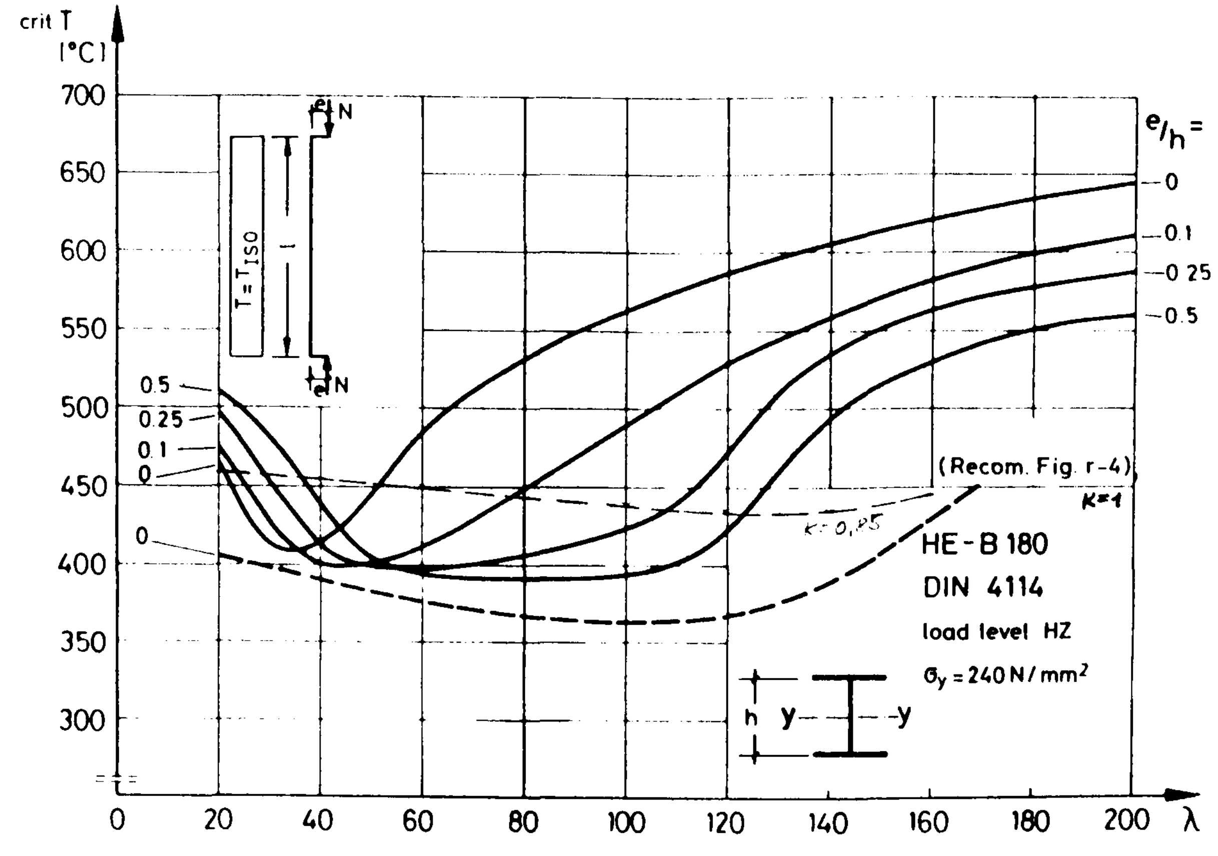



Fig. 8: Critical temperature of steel columns - buckling about the strong axis - depending on load eccentricity

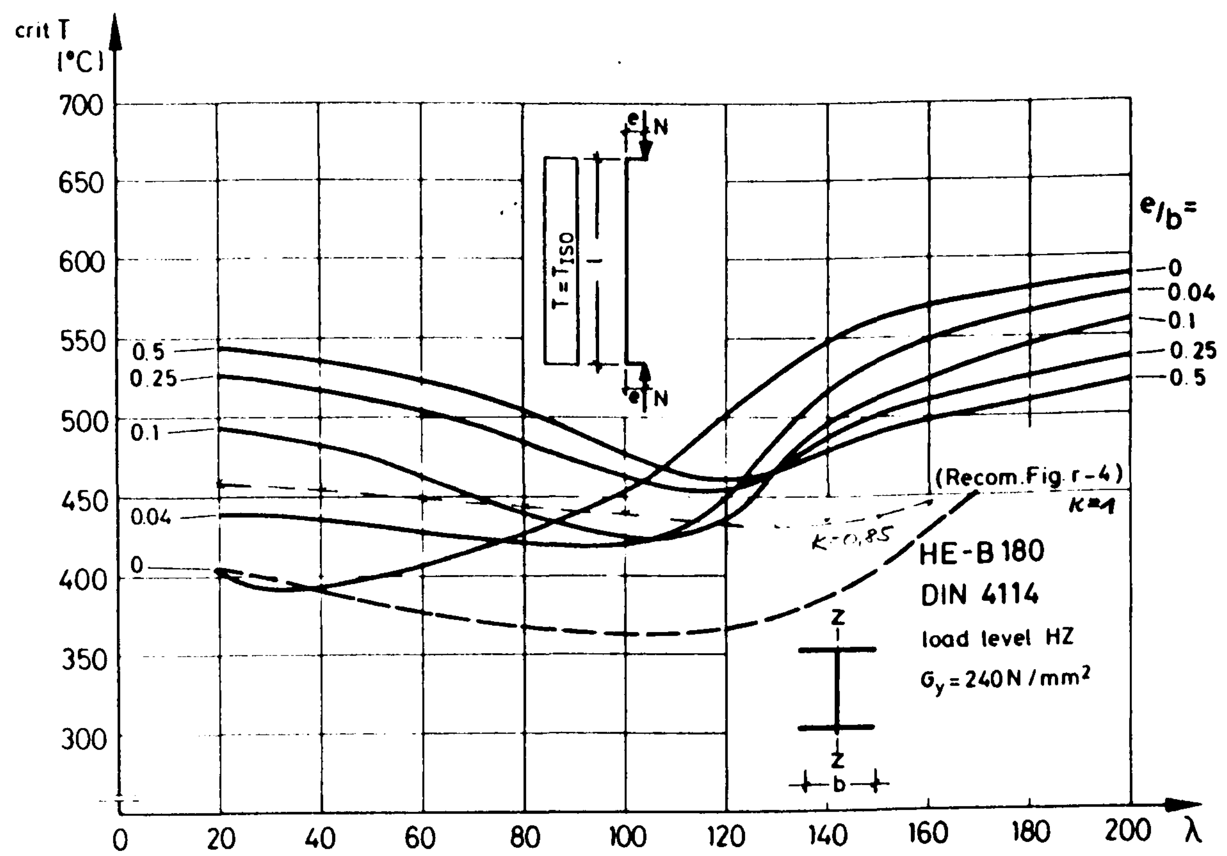
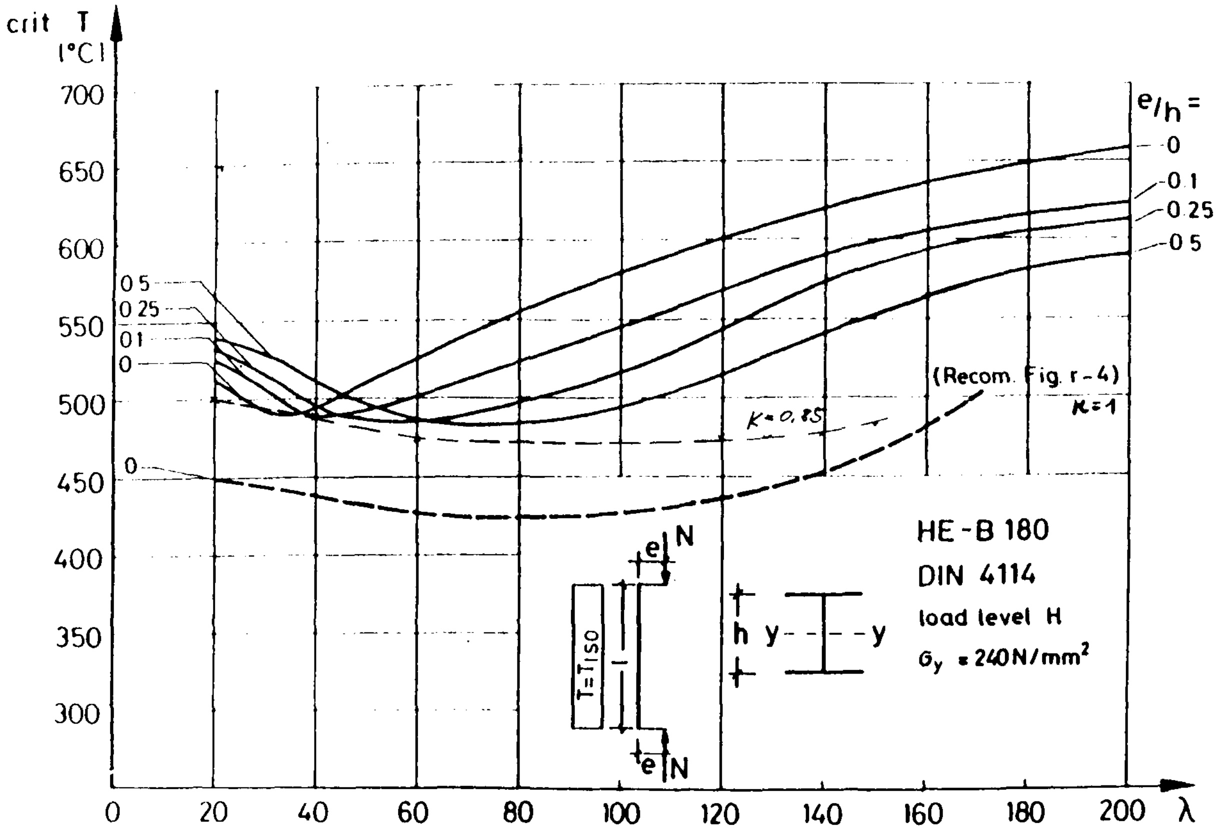
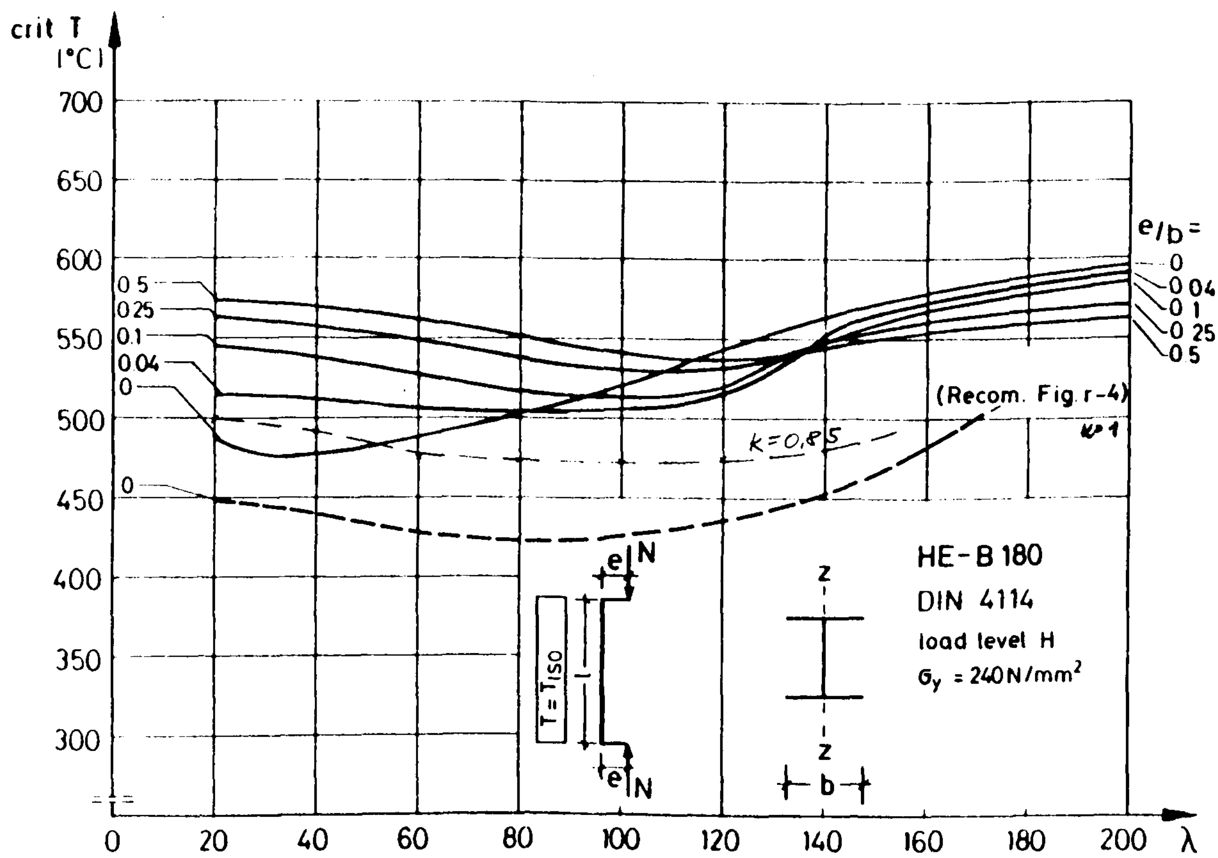



Fig. 9: Critical temperature of steel columns - buckling about the weak axis - depending on load eccentricity




<u>Fig. 10:</u> Critical temperature of steel columns - buckling about the strong axis - depending on load eccentricity




<u>Fig. 11:</u> Critical temperature of steel columns - buckling about the weak axis - depending on load eccentricity

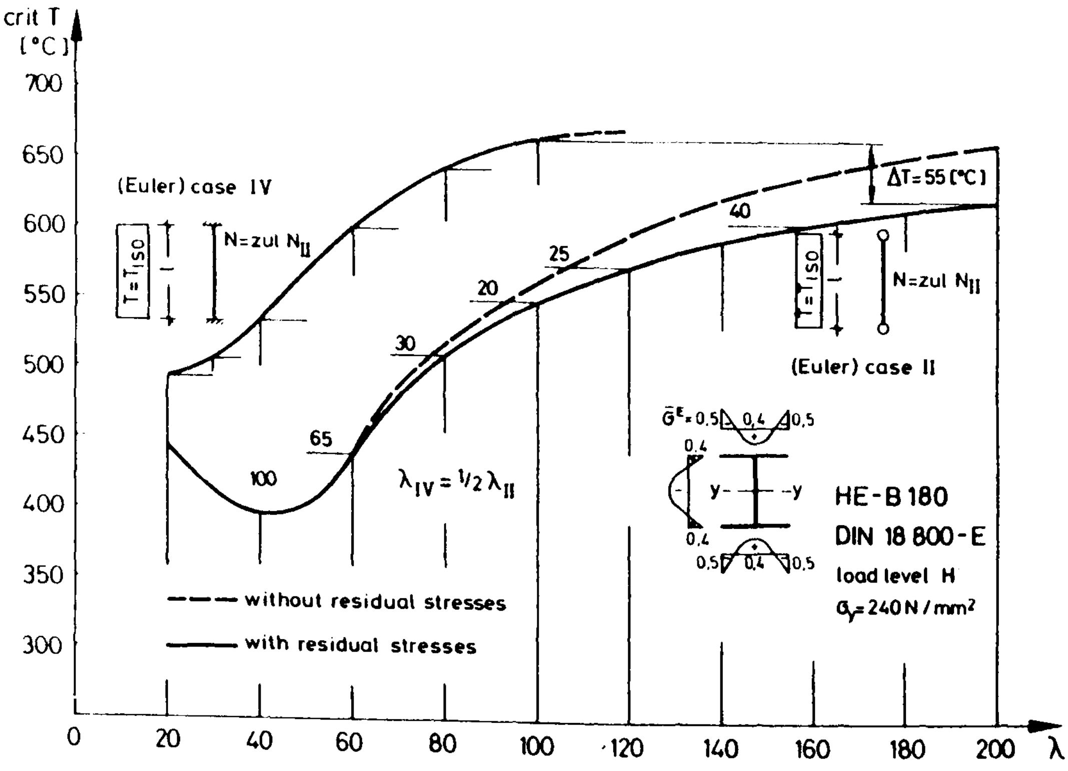



Fig. 12.: Critical temperature of steel columns - buckling about the strong axis - taking into account residual stresses of two loading rates

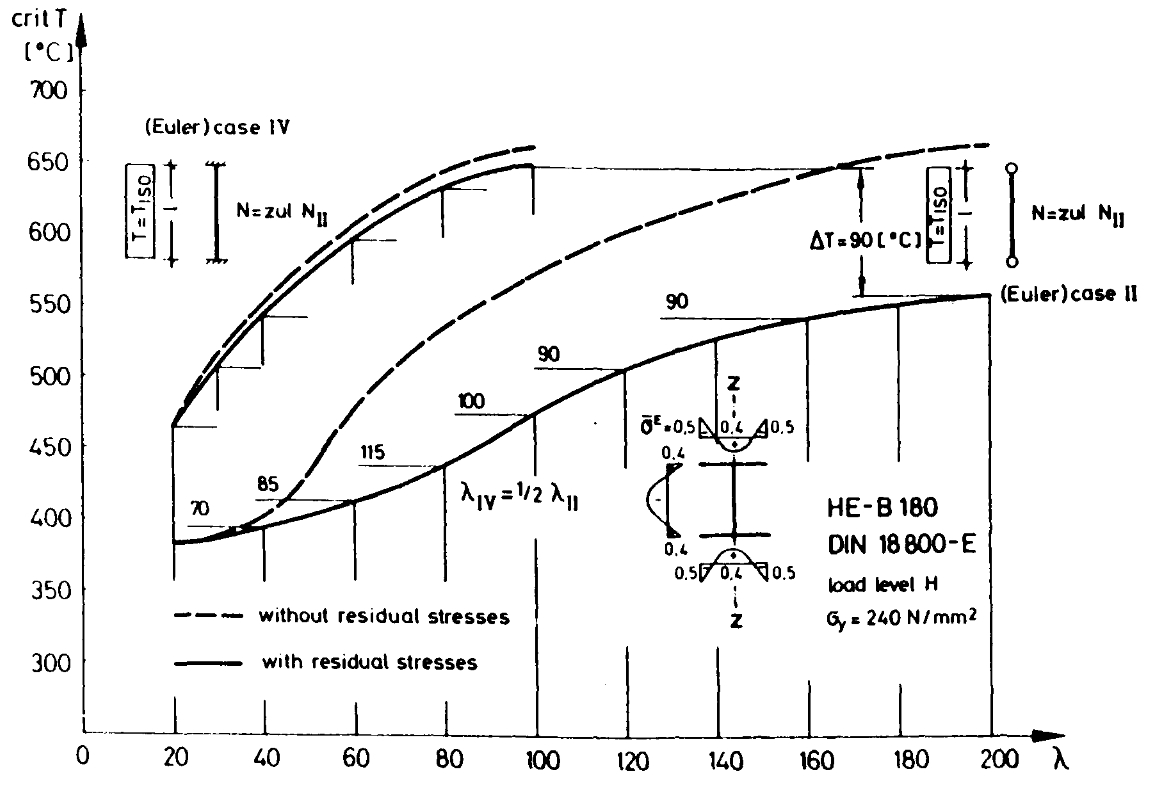



Fig. 13: Critical temperature of steel columns - buckling about the weak axis - taking into account residual stresses of two loading rates

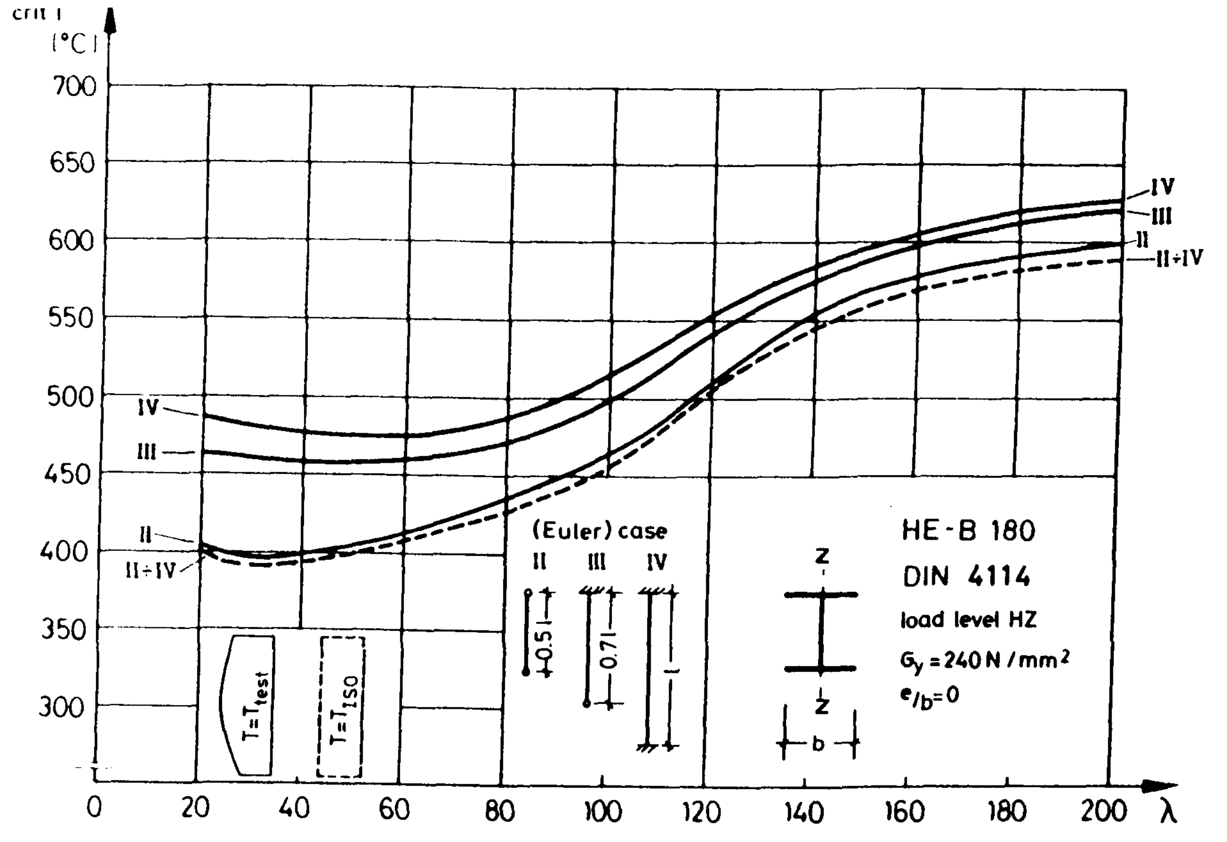



Fig. 14: Critical temperature of steel columns - buckling about the weak axis - for different boundary conditions depending on the temperature distribution along the column

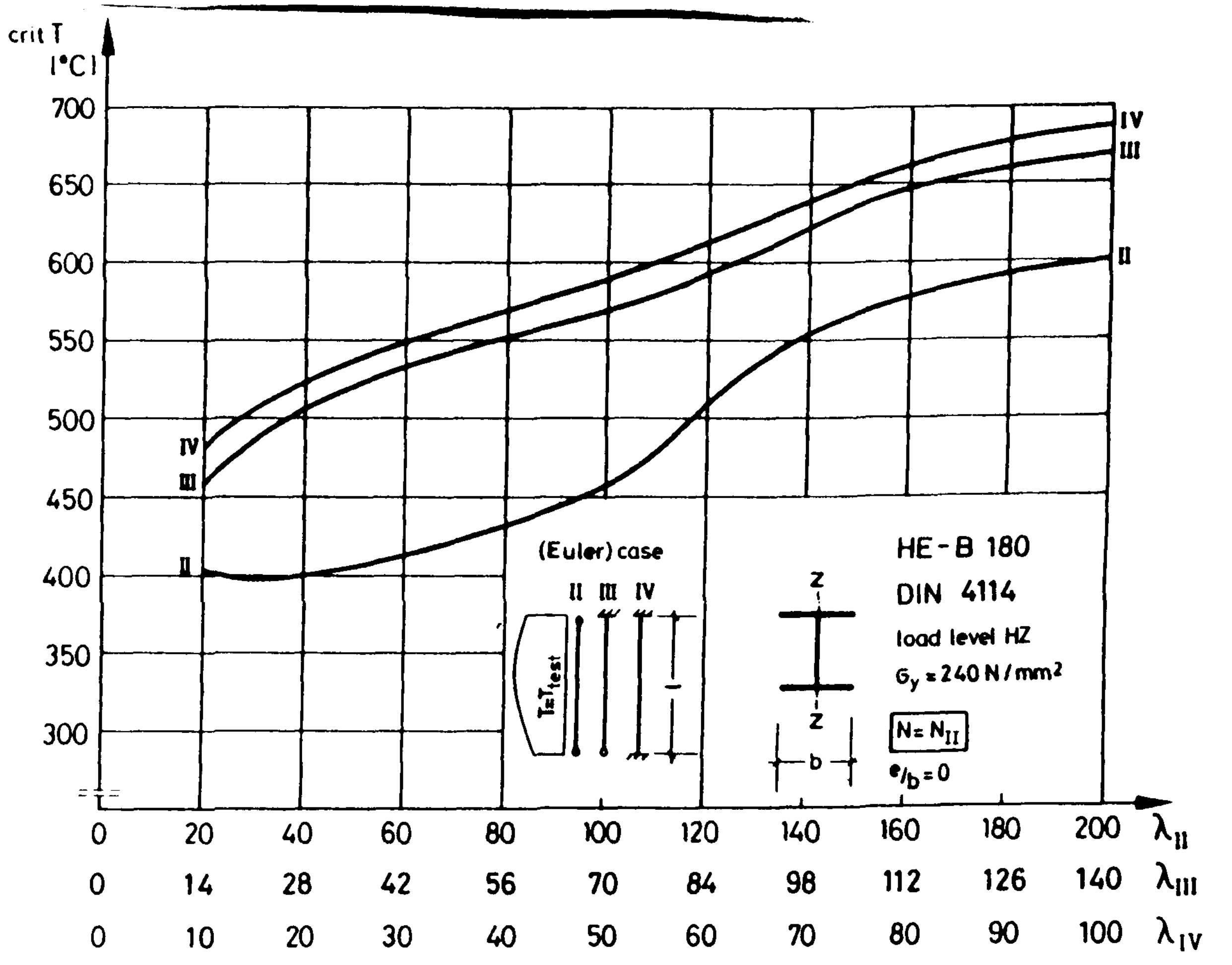



Fig. 15: Critical temperature of steel columns - buckling about the weak axis - for different boundary conditions taking into account the design load (case II) and a temperature distribution along the column

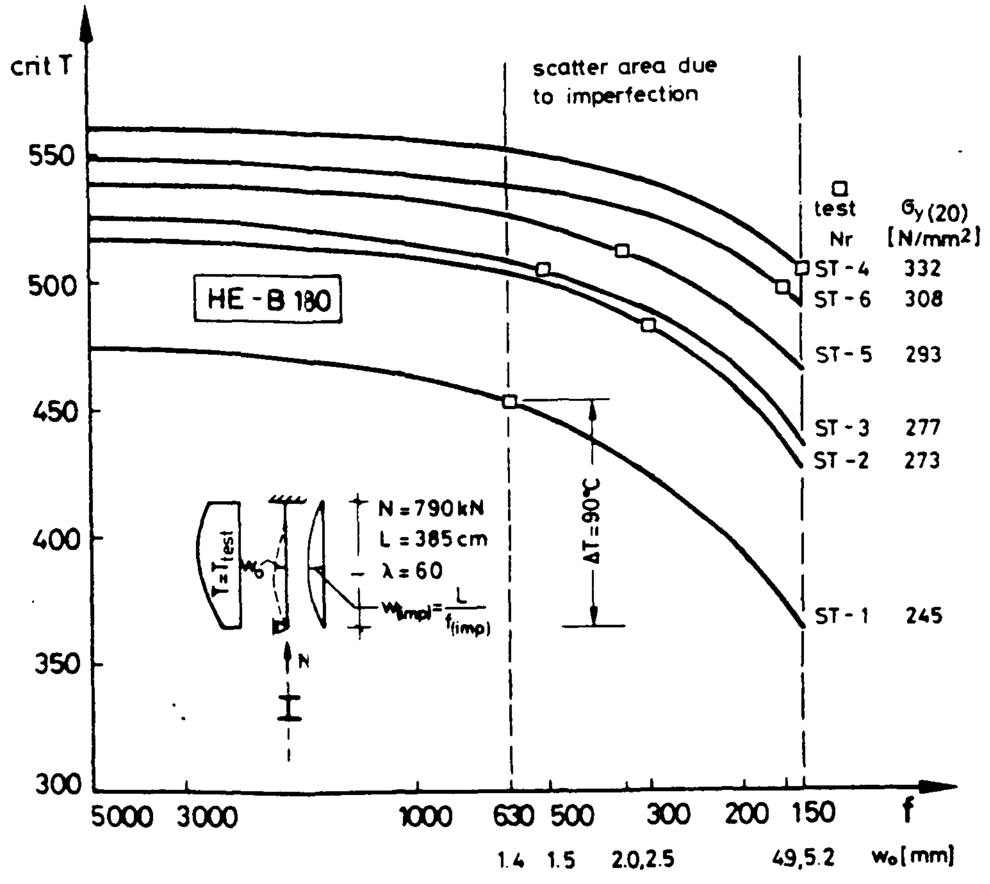



Fig. 16: Influence of imperfections of the critical temperature for centrical loaded steel columns

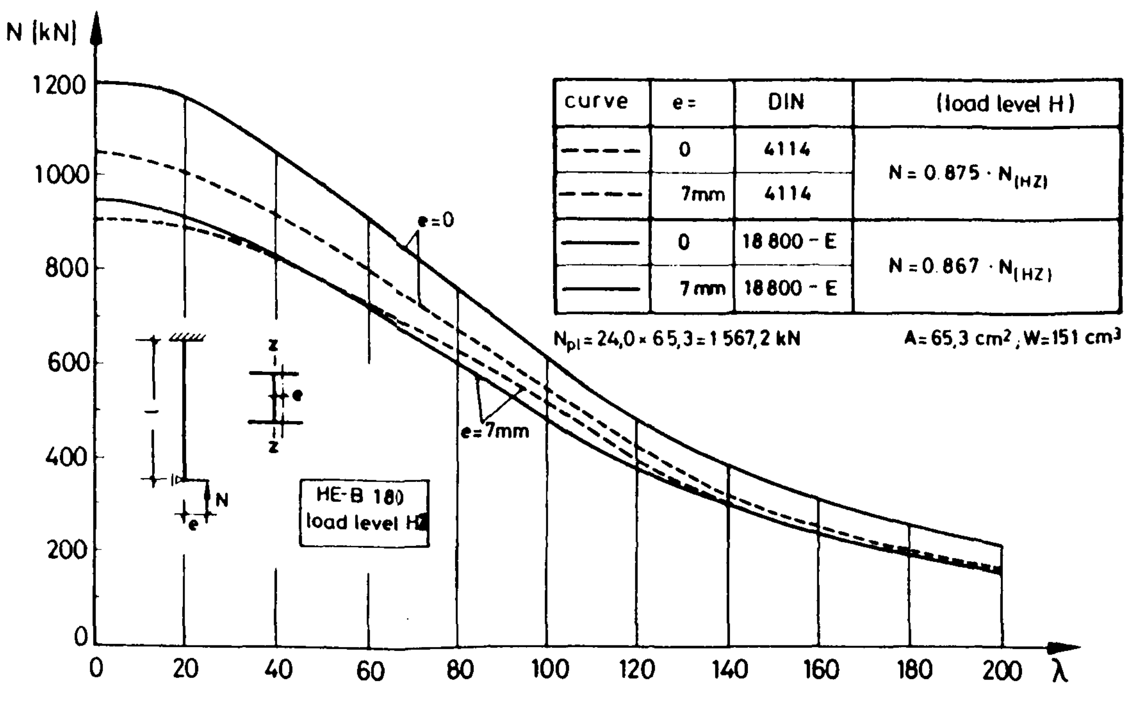



Fig. 17: Design load according to the actual German Code DIN 4114 and the draft of the new German Code DIN 18 800 (based on European buckling curves)

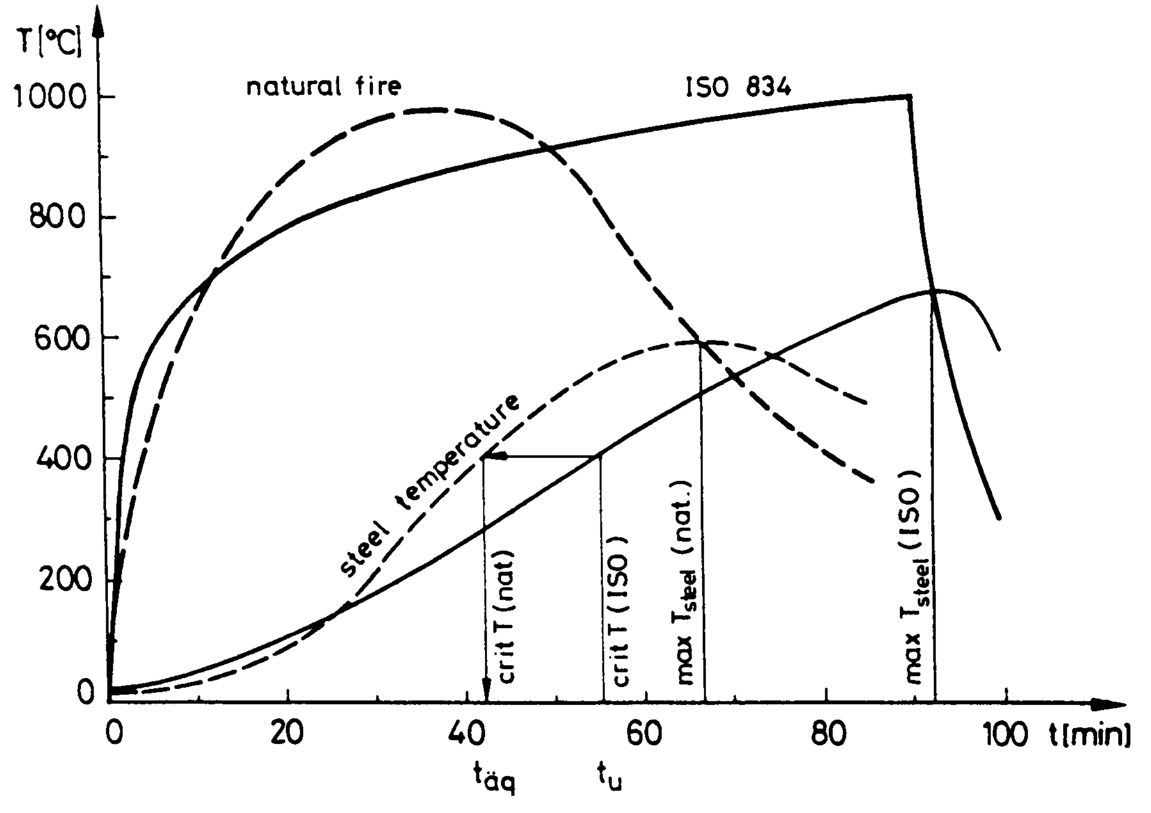



Fig. 18: Relation between failure times under ISO 834 fire conditions and natural fire conditions

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